SEISMIC STRENGTHENING OF COLUMNS BY ADDING NEW CONCRETE

Stephanos E. Dritsos[1]

ABSTRACT

Placing reinforced concrete jackets or layers to strengthen or repair and strengthen concrete columns is a normal construction practice but there are many unresolved issues regarding the capacity of the strengthened elements. In the absence of any guidance, engineering judgement is often used. This paper sets out to assist the engineer when considering some of these unresolved issues. Revised values for factors of safety are proposed for design. A procedure to guarantee a sufficient connection between contact surfaces and to determine the performance of retrofitted columns is presented, considering the strengthened columns as "composite" elements. The parameters affecting the main mechanisms for the transfer of shear stress at the interface between new and old concrete are described and practical design considerations are given. An approximate procedure is presented, based on the design of monolithic elements, supplemented by the use of specific modification factors (monolithic factors), in order to evaluate the capacity of a strengthened element. Available experimental results are processed to derive appropriate values for monolithic behaviour factors and an extended analytical analysis is used to fill in gaps in the experimental work. Although this paper has particular relevance to seismic strengthening, its contents will have a wider application to strengthening in general. The object of this paper is to provide guidance so that the engineer is better equipped to deal with the practical design needs of today.

Keywords: Column, Concrete, Earthquake, Jacket, Redesign, Repair, Retrofitting, Seismic and Strengthening.

1. INTRODUCTION

It is now recognized in most earthquake prone areas that the vast majority of the existing building stock is under a much higher seismic risk when compared to that of new buildings. This is because old buildings were constructed either before the implementation of a seismic risk Code or under the provisions of old seismic Codes, which are now known to be inadequate. In Greece, the proportion of old buildings at risk is about 80% when considering 1985 as the year to distinguish between old and new buildings (a major revision of the existing seismic Code was implemented in this year). Therefore, seismic strengthening is a requirement in many cases. The selection of the most suitable strengthening technique for a specific building is a difficult issue. The engineer must decide between a number of alternative strategies and methods, which may be unfamiliar or may need specialist staff for implementation.

For existing reinforced concrete buildings, seismic strengthening normally focuses on the vertical elements (columns and walls). Strengthening of beams is usually considered as a second priority, since the structural integrity is mainly affected by the capacity of the vertical elements and the technique, when executed for beams, may disturb the residents of the upper level in a multi-storey building. A number of techniques, in the form of jacketing, are used in practice to strengthen deficient reinforced concrete columns. Concrete, steel or fiber reinforced polymers (FRP) are normally used for the jacket.

Steel and FRP jacketing are very popular in many countries since they have the advantage of a minimal increase of cross sectional dimensions of the columns. Moreover, they can be executed quickly with little interruption to the use of the structure while work is carried out. Extensive research results for both the above techniques have already been published. For steel jacketing, the pioneering experimental research undertaken at the University of California, San Diego (Chai *et al.*, 1991; Chai *et al*., 1994; Priestley *et a*l., 1994a; Priestley *et al*., 1994b;) should be mentioned while hundreds of papers have been presented on FRP jacketing. A comprehensive literature review on relevant publications can be found in fib Bul. N$^{o.}$ 14 (2001) and fib Bul. N$^{o.}$ 24 (2003). The above techniques are normally used to enhance column ductility and shear capacity and are very effective in preventing bond failure in columns with inadequately lapped longitudinal reinforcement. However, they offer little to the axial and flexural strength of an element and are inappropriate if a considerable increase in stiffness is required. If this is the case, concrete jacketing has the advantage. Moreover, in many countries such as Greece, where reinforced concrete is the most popular construction material for new buildings, engineers appear to prefer the strengthening solution of adding new concrete. This preference is because engineers are familiar with this type of construction, while local experienced contractors and personnel are readily available.

[1] *Associate Professor of the Department of Civil Engineering at the University of Patras, Patras, Greece (Member).*

Concrete jacketing has been experimentally investigated in the past. Bett *et al*. (1988), Rodriguez and Park (1991), Ersoy *et al*. (1993), Rodriguez and Park (1994), Hakuto (1995), Stoppenhagen *et al*. (1995) Dritsos *et al*. (1997), Gomes and Appleton (1998), Rodriguez and Santiago (1998), Julio *et al*. (2003), Tsonos (2004) and Julio *et al.* (2005) have presented interesting research and it has been proved that the bending, the shear capacity, the stiffness and the ductility of columns can be improved. The method can also improve the axial load carrying capacity of columns and may alleviate problems caused by inadequate lap splice lengths (Bousias *et al.*, 2004). If site conditions do not allowed the construction of jackets, strengthening can be performed by placing an addition concrete layer on one or more sides of the element.

Although placing reinforced concrete jackets or layers is a normal construction practice and a number of papers have been presented, there are still many unresolved issues concerning the evaluation of the capacity of these strengthened elements. The scope of this paper is to answer some of these unresolved issues. A lack of knowledge often leads to an unreasonable use of the technique, high costs and, at times, unproductive results. This paper, in part, follows on from a previous work (Dritsos 2005a), where the practical applications of strengthening or repair and strengthening of buildings before and after an earthquake were investigated. This paper concentrates on column concrete jacketing.

Strengthened or repaired and strengthened elements from reinforced concrete can be considered as, in general, multi-phased elements. They consist of the initial parts or components of the load bearing structure and new elements (made from similar or other materials) that are connected in order to limit shear slippage between the contact surfaces and to avoid detachment. Consequently, for the structural design of the above elements, the process of designing composite elements should be followed and the transfer mechanism of the forces at the interface between the old and the new element should be taken into consideration.

Obviously, the evaluation of the capacity of strengthened or repaired and strengthened elements does not have the same degree of reliability as that for monolithic elements of new structures. The reliability of calculations decreases because there are more uncertainties. These uncertainties are due to the following:

(a) A lack of sufficient documentation and exploitable scientific knowledge in the field of contact surface mechanics with regard to the distribution of stress in the old and the new element (this would include existing gravity loads on the initial element, any remaining deformities or any potential unloading),

(b) The accuracy of the assessment of the capacity of damaged elements, as it has usually been evaluated using empirical or semi-empirical methods. That is, since damage cannot be measured accurately, its assessment is influenced by engineering judgement and

(c) The details of the implementation of the work and the quality control influence drastically the effectiveness of the intervention and, consequently, the behaviour of the strengthened or repaired and strengthened elements.

The above uncertainties should be taken into consideration, when evaluating the resistance of a retrofitted element, by means of special partial safety factors (γ_{Rd}). However, for critical projects, analytical results should be confirmed with laboratory trials (GRECO, 2005).

The structural design of strengthened or repaired and strengthened concrete elements can be placed into the framework of the presently known processes of design that are used for new constructions, supplemented by the following (Tassios, 1983; EC 8, 1995; GRECO, 2005; Dritsos, 2005b):

(a) New revised factors of safety for old and new materials are used,

(b) The interface between the contact surfaces should be investigated to ensure that, by calculation, failure in each strengthened or repaired and strengthened element precedes failure at the interface between the old and the new material and

(c) The mechanical characteristics of each strengthened or repaired and strengthened element should be determined by considering the elements as composite elements, by taking into account the slippage at the interface between the existing and the new element. Alternatively, an approximate process involving monolithic correction factors (monolithic behaviour coefficients) could be used.

The above three critical points for the structural design of strengthened or repaired and strengthened elements will be described in detail in the following sections.

2. REVISED FACTORS OF SAFETY

If in an existing structure, the dimensions of elements have been measured, the location and size of the reinforcement have been determined and the individual strengths of the existing materials have been established, the actual strengths of the elements, ignoring gross constructional faults, may be greater than that of the initial design. Therefore, under certain conditions, lower partial safety factors for the materials of the structural system could be proposed. Partial safety factor values of γ_c equal to 1.2 for concrete and γ_s equal to 1.05 for steel are proposed in Part 1-4 of EC 8 (1995). In the most recent draft of GRECO (2005), values for γ_c and γ_s depend on the reliability level of the documentation procedure that is applied to assess the mechanical characteristics of the existing materials. For the highest reliability level, the proposed values of γ_c and γ_s are 1.35 and 1.05 respectively. For the medium reliability level, the proposed respective values of 1.50 and 1.15 are the same as those for new structures, while for the lowest reliability level they equal 1.65 and 1.25 respectively. As far as dead load partial safety factors (γ_g) are concerned, values should also depend on the reliability level of the procedure used to assess the dead loads of a structure. The GRECO (2005) proposed values for γ_g are 1.20, 1.35 and 1.50 for the high, medium and low reliability levels respectively. Again, the proposed medium reliability level value is the same as that for new structures. If in design, strength reduction factors are used instead of partial safety factors, as is the practice in many countries outside Europe, the above considerations could be expressed as follows: for a medium realibility level to evaluate existing loading and material characteristics, strength reduction factor values could be considered the same as that for new structures. If the documentation procedure for the evaluation of material strength and existing load leads to a higher or lower reliability level, a respective increase or reduction of the above values by 10% should be considered.

For new materials added during an intervention, partial safety factor values are generally larger than those specified for new structures. This would be because the repair work may often be carried out under difficult conditions of access and unknown levels of quality control and supervision and, therefore, the uncertainty of achieving the desired strength can be greater. For concrete and steel reinforcement, GRECO (2005) advises applying multiplication factors of 1.10 or 1.20 (depending on the construction conditions) to the partial safety factors of 1.50 and 1.15 for γ_c and γ_s respectively, which are adopted for new structures. Taking into account that the multiplication factors of 1.10 and 1.20 concern only material strength and not loads, strength reduction factors, when used, should be divided by 1.05 and 1.10 respectively.

3. CONTROL OF A SUFFICIENT CONNECTION BETWEEN CONTACT SURFACES

Load transfer mechanisms between the old and new materials must be capable of transferring the tensile, compressive and shear stresses that develop at the interface.

As far as interface tensile stresses are concerned, the transfer can be guaranteed if the developed stresses are lower than the tensile strength of the weakest concrete. If not, an appropriate quantity of reinforcement or anchor bars crossing to the contact surface should be provided, as specified later in this paper.

Regarding concrete-to-concrete direct compression, a full continuity compression transfer can be expected at the interface if adequate treatment measures have been performed on the old concrete surface (such as roughening). However, a lower modulus of elasticity should be considered for concrete adjacent the interface, as higher deformations develop due to the mechanical treatment of the existing concrete and contact and compaction imperfections (CEB Bul. N°. 162, 1983). Obviously, the interface compressive strength can be considered to equal the lowest compressive strength of the contact materials.

In order to guarantee a sufficient connection between contact surfaces, the check for safety at the ultimate limit state can be expressed symbolically by the following equation of safety:

$$S_d \leq R_d \tag{1}$$

where: S_d is the design action effect and R_d is the design resistance. This control will include checking the shear force and the shear resistance at the interface between the old and the new element. That is to say, the following relationship must be satisfied:

$$V_{Sd}^{interface} \leq V_{Rd}^{interface} \tag{2}$$

where: $V_{Sd}^{interface}$ is the shear force acting at the interface and $V_{Rd}^{interface}$ is the shear resistance at the interface.

Obviously, a guaranteed connection that avoids premature failure would be desirable. This would be because it represents the critical factor for the effectiveness of the intervention and would ensure an acceptable degree of reliability for calculations.

If failure between the contact surfaces precedes failure of the strengthened element, the load bearing capacity of the connection will determine the load bearing capacity of the strengthened element. In addition, the load bearing capacity of the strengthened element cannot be considered smaller than that of the original unstrengthened element.

The control between contact surfaces along the whole length of the strengthening structural element should be based on average values of $V_{Sd}^{interface}$ and $V_{Rd}^{interface}$ corresponding to various segments of length l_{i-j} (i and j for successive segments) into which the element has been divided. That is to say:

$$V_{Sd(i-j)}^{interface} \leq V_{Rd(i-j)}^{interface} \tag{3}$$

The length of segments should not be greater than twice the width of the cross section of the column. Furthermore, following the same rules adopted for the design of composite elements (for example, EC 4, 2002), the process can be facilitated if segments breaks are located at characteristic cross sections. As such, sections dividing an element should be placed at the following locations: (a) at the largest positive or negative bending moment, (b) at the supports, (c) at positions of point loads, (d) where there are abrupt changes in cross section and (e) at the ends of cantilevers.

3.1 Shear forces acting at the interface

An evaluation of the shear force that develops between the contact surfaces ($V_{Sd(i-j)}^{interface}$ from equation 3) can be obtained by analysing each segment assuming monolithic behaviour (by approximately calculating the shear stress at the interface using mechanics theory). Alternatively, the more accurate calculation method that is applied for steel and concrete composite structural elements could be used. Figure 1 schematically illustrates the shear force that develops between contact surfaces for the three cases where concrete is added as an under layer, an over layer or a side layer. If a structural element has been strengthened with the new layer of concrete, the size of the shear resistance between the contact surfaces, for a segment length of l_{i-j}, can be determined by considering the equilibrium of forces in the new concrete segment ABCDA of figure 1. That is:

$$V_{Sd(i-j)}^{interface} = V_{Sd}^{BC} = F_{AB} - F_{CD} \tag{4}$$

A process of section analysis can be used to determine the magnitudes of the forces F_{AB} and F_{CD}. That is, by taking sections through the whole element at positions i and j respectively and determining the internal tensile or compressive forces corresponding to layer sections AB or CD.

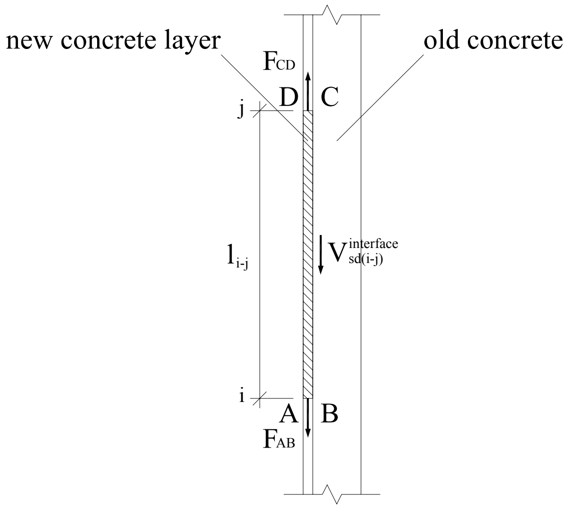


Figure 1. Shear force at the interface

3.2 Interface shear resistance

Four mechanisms contribute to the shear resistance at the interface ($V^{interface}_{Rd(i-j)}$ from equation 3 above). These are concrete-to-concrete adhesion, concrete-to-concrete friction, the connecting action from either steel bars placed across the interface between the old and the new concrete or bent down bars welded between the bars of the old and the new concrete. These four mechanisms can be subdivided into the two groups of unreinforced and reinforced interfaces, depending on whether or not additional steel is placed across the interface or welded between the bars of the old and the new concrete. In general, the shear resistance developed at the interface depends on the amount of slippage at the interface.

3.2.1 Unreinforced interfaces

The two mechanisms acting at an unreinforced interface are adhesion and friction. It must be noted that maximum adhesion values are achieved for low interface slip values (in the region of 0.02 mm), while friction becomes important for much higher slippages. Therefore, the maximum resistances from adhesion and friction do no coincide and cannot be considered to act together.

Figure 2 (CEB Bul. N$^{o.}$ 162, 1983) presents shear resistance (τ) against slip (s) plots for some cases of original column surface roughness with and without adhesion. Factors that affect the adhesion between existing and new concrete are the tensile strength of the contact materials, the surface roughness of the original column (as demonstrated by figure 2) and the surface treatment of the original column (exposing the aggregate gives higher bond strengths). Moreover, the method of placing the new concrete has an effect (shotcrete is better than in-situ concrete because the impact forces mortar into surface pores and voids). In addition, monolithic behaviour can be expected when bonding agents are properly applied between the contact surfaces.

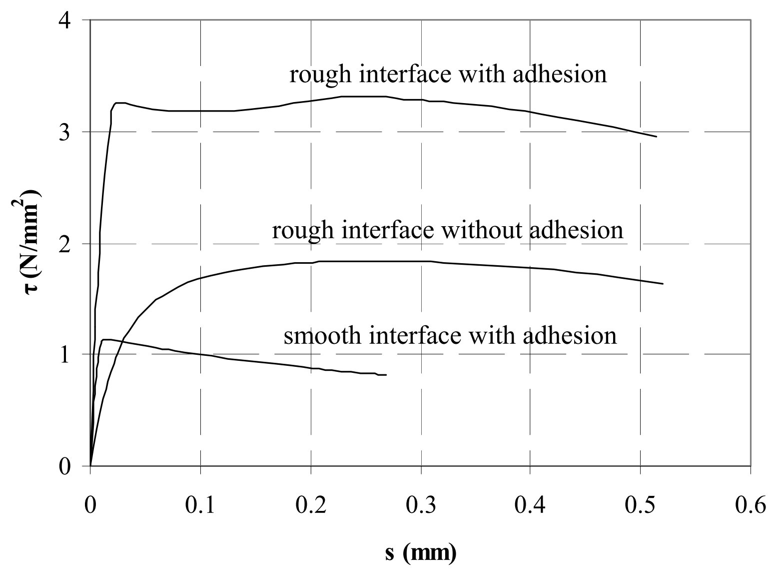


Figure 2. Concrete-to-concrete adhesion (CEB Bul. N$^{o.}$ 162, 1983).

The parameters that affect concrete-to-concrete friction are the size and shape of the aggregates if exposed (large angular aggregates are better) and the surface roughness of the original column (rougher surfaces have greater areas of surface contact). Additional parameters include the concrete compressive strength, the external normal compressive stress (a higher normal stress gives a higher shear stiffness) and if the loading is cyclic or not (cyclic loading quickly deteriorates the contact surfaces giving a larger slip or a lower shear response). Representations that model concrete-to-concrete friction can be found in the literature (CEB Bul. N$^{o.}$ 162, 1983) and figure 3 presents a model proposed by Tsoukantas and Tassios (1989). From figure 3, it can be seen that the shear resistance due to friction (τ_f) reaches a maximum when the relative slip is in the region of 1.75 mm. The maximum value of the design concrete-to-concrete shear resistance due to friction (τ_{fu}) can be calculated from the following equation:

$$\tau_{fu} = 0.4(f_c^2 * \sigma_c)^{1/3} \tag{5}$$

where: f_c is the compressive strength of the weaker concrete and σ_c is the interface compressive stress.

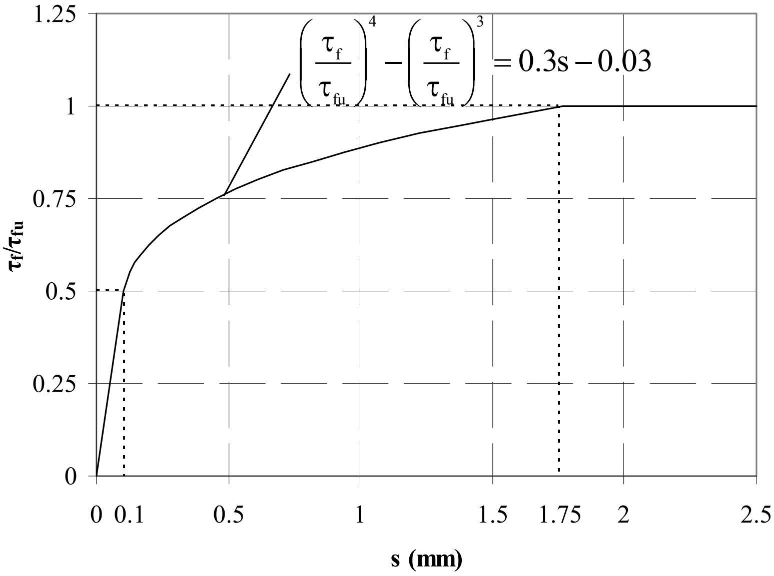


Figure 3. Roughened interface concrete-to-concrete friction (Tsoukantas and Tassios, 1989).

3.2.2 Reinforced interfaces

When a steel bar crosses the interface between old and new concrete, an additional action that may occur is clamping action. This action would take place when the surface of the old concrete has been roughened, or shotcrete has been placed and if the steel bar is adequately anchored. When a shear stress is applied, a slip is produced and the contact surface between the old and the new concrete must open as one surface rides up over the other due to the roughness. Therefore, a tensile stress is activated in the steel bar, which in turn produces a corresponding compressive stress, or clamping action, and a frictional resistance is mobilised. Equation 5 can be modified in order to take into account the additional frictional resistance mobilised by clamping action, as follows:

$$\tau_{fu} = 0.4(f_c^2 * (\sigma_c + \rho_d f_y))^{1/3} \tag{6}$$

where: ρ_d is the total cross sectional area of the shear connectors (A_{sd}) divided by the cross sectional area between the contact surfaces (A_{cd}).

Figure 4 (CEB Bul. N$^{o.}$ 162, 1983) presents a plot of normalised shear resistance (V) against slip for dowel action for monotonic loading. Parameters that affect dowel action include the concrete strengths of the new and the old concrete, the yield stress of the dowel (f_y), the diameter of the dowel (d_b) and the amount of dowels placed. Representations such as figure 4 can only be used if the dowels are adequately embedded in the old and the new concrete (at depths of at least 6 times the dowel diameter). In addition, measures should be taken to avoid failure due to placing dowels too close to the edge of the concrete (at least 3, 5 or 6 times the dowel diameter are respectively required from the edge of the original column or the top or base of the original column or jacket if a partial jacket is placed). The maximum value of the design shear resistance from dowel action (V_u) can be calculated from the following equation (Rasmussen, 1963; Vintzeleou and Tassios 1986):

$$V_u = 1.3 * d_b^2 * \sqrt{f_c * f_y} \tag{7}$$

If earthquake action is expected, it would be conservative to remove the value of 1.3 from equation 7 (GRECO, 2005).

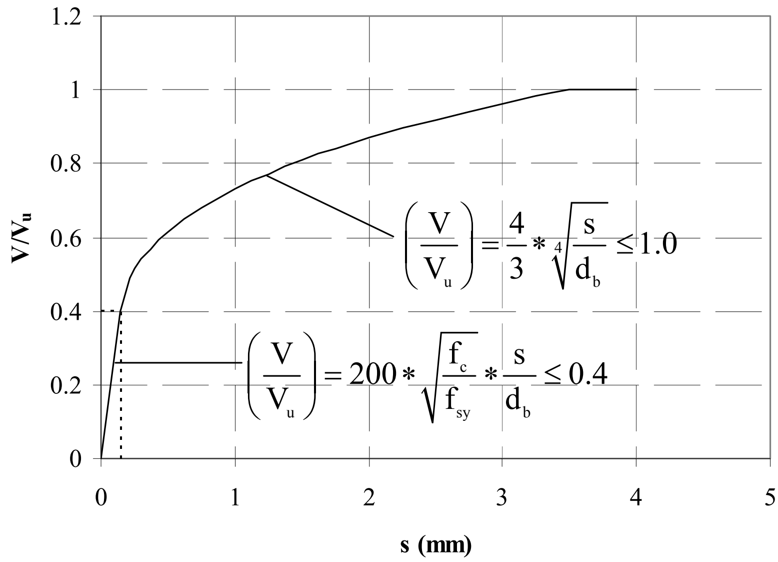


Figure 4. Shear force against slip distribution for dowel action (CEB Bul. N$^{o.}$ 162, 1983).

A practice that is commonly used and has a good reputation is to weld bent down bars between the reinforcement of the old concrete and the new concrete. When there is relative slip between the old and the new concrete, a part of the force in the old bar is transferred to the new bar via the bent down bar. Figure 5a conservatively demonstrates the mechanism (CEB Bul. N$^{o.}$ 162, 1983; Tassios, 2004; Tassios, 2005). When there is slippage at the interface, one of the angled legs of the bent down bar is elongated by a factor of s/(√2) while the other angled leg is shortened by the same factor. Therefore, the respective tensile or compressive strains (ε_{sb}) and stresses (σ_{sb}) are:

$$\varepsilon_{sb} = \frac{s/\sqrt{2}}{\sqrt{2}h_s} = \frac{s}{2h_s} \text{ and } \sigma_{sb} = E_s \frac{s}{2h_s} \leq f_{yb} \tag{8}$$

where: h_s is the distance between the centrelines of the outer arms of the bent down bar, E_s is the modulus of elasticity for the steel bar and f_{yb} is the characteristic value of the yield strength of the steel bar. By considering the equilibrium of forces, the following equations can be derived:

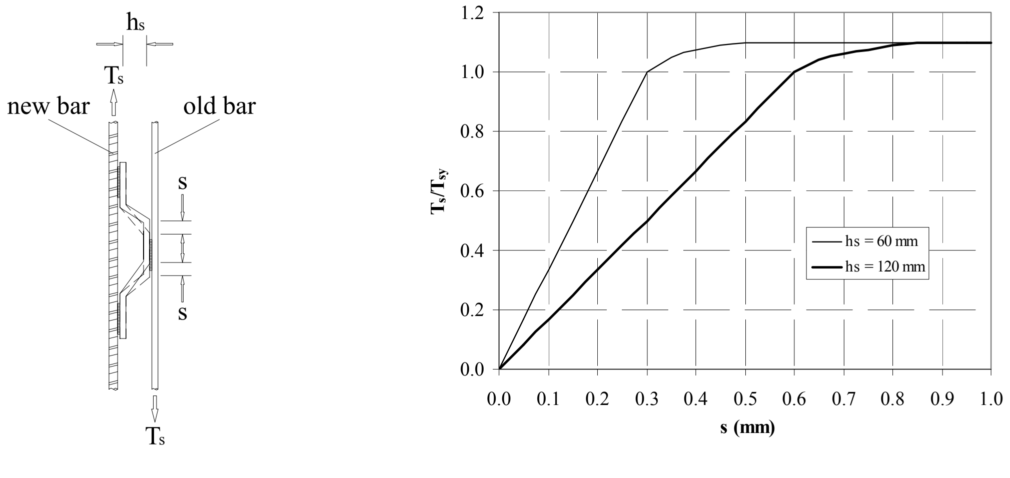
$$\frac{T_s}{\sqrt{2}} = A_{sb}\sigma_{sb} = A_{sb} * E_s \frac{s}{2h_s} \leq A_{sb}f_{yb} \tag{9}$$

and

$$T_s = A_{sb} * E_s \frac{s}{\sqrt{2}h_s} \leq T_{sy} = \sqrt{2}A_{sb}f_{yb} \tag{10}$$

where T_s is the force that can be transferred to the new reinforcement, expressing in other words the shear capacity of the interface, A_{sb} is the cross sectional area of the bent down bar and T_{sy} is the force required to yield the bar.

Figure 5b presents plots of T_s/T_{sy} against slip for grade S500 steel bent down bars for distances of h_s of 60 mm and 120 mm, where strain hardening of the steel has been taken into consideration. It can be seen from figure 5b that the mechanism is mobilised for very small slippages and this justifies the well-deserved reputation of bent down bars.



(a) (b)

Figure 5. (a) Bent down bar model and (b) normalised force against slippage.

3.2.3 Design considerations

The total shear resistance between contact surfaces can be found by summing the individual shear resistances that are mobilised by each individual mechanism for a common interface slip. Figure 6a presents a plot of the superposition of slippage from all the mechanisms discussed above for the transfer of shear stress at the interface and figure 6b presents linear simplifications of the longitudinal transfer of shear stress. As can be seen from figure 6a, the problem becomes complicated when all the mechanisms are considered to act together. When considering the required performance level, if an acceptable value of slippage is determined, the respective interface resistance can be found by calculating the resistance for each mechanism and summing the results. Alternatively, in order to simplify calculations, bilinear diagrams of the type OY_0U of figure 6b could be applied. Elastic simplifications, as in curves OY_1 or OU of figure 6b, could be used to facilitate the analysis. More precise results could be obtained by using elasto-plastic diagrams such as curve OY_1UR of figure 6b. In general, the remaining shear resistance (V_{res}) could be considered as insignificant. In figure 6b, the ultimate interface shear resistance (V_u) is defined as being 20% less than the maximum shear resistance (V_{max}) and the failure slip (s_u) corresponds to this ultimate shear resistance.

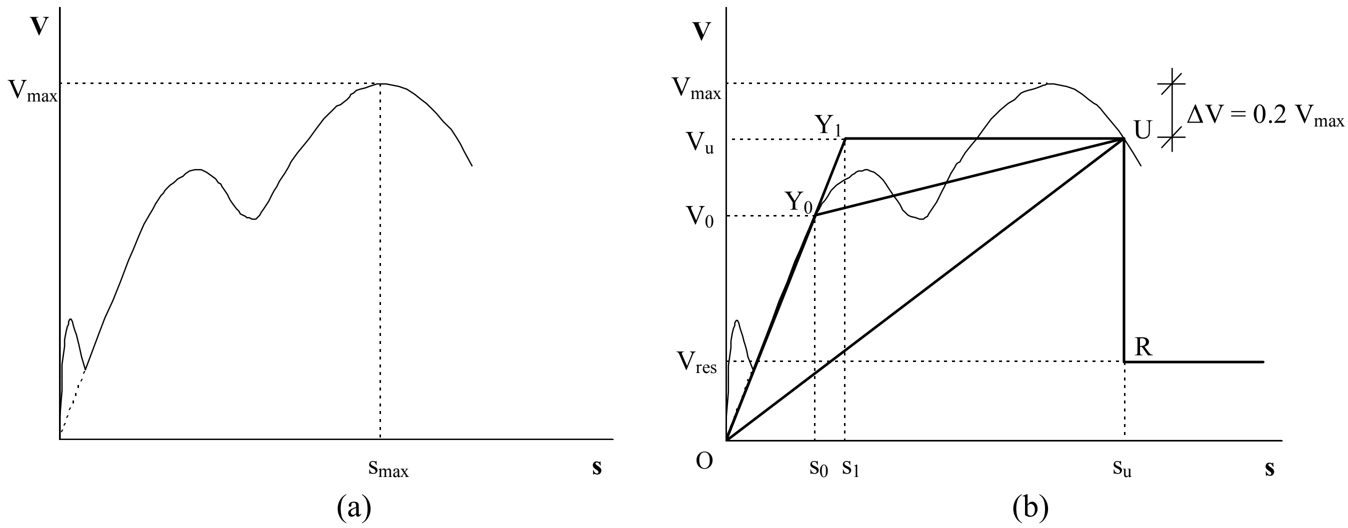


Figure 6. The longitudinal transfer of shear stress (a) superposition of mechanisms and (b) linear simplifications.

For structural elements that resist seismic actions, it may be useful (and it would simplify calculations) if the mechanisms of adhesion and friction were ignored and only the shear resistance from dowels or other shear connectors is taken into consideration. In other elements that do not resist seismic action (for example concrete slabs), it could be considered that shear connectors are required only when, in some region of the structural element, the shear stress between the contact surfaces exceeds the shear strength from adhesion or friction. It is conservatively acceptable to say that if failure occurs between the contact surfaces at some point and a crack appears, the crack will extend to the remaining region between the contact surfaces even if the shear force has been calculated to be less than the corresponding shear resistance.

In order to prevent a brittle failure at the interface, a minimum amount of steel shear connectors in the form of dowels or bent down bars are required for concrete-to-concrete connections. The required percentage can be calculated in a similar way to that of determining the minimum shear reinforcement in monolithic elements and the following relationship has been proposed (Dritsos, 2005b; GRECO, 2005):

$$\rho_d \geq \max\,(0.18\, f_{ctm}/f_{yk},\ 0.12\%) \qquad (11a)$$

where: f_{yk} is the characteristic yield strength of the steel shear connectors or bent down bars and f_{ctm} is the average tensile strength of concrete.

The average tensile strength of concrete can be evaluated through the compressive concrete strength by considering available literature or code relationships. In EC 2 (2004), the average tensile strength, for concrete grades less than 50 MPa, is expressed as:

$$f_{ctm} = 0.3\, f_{ck}^{2/3} \text{ (MPa)} \qquad (11b)$$

4. CAPACITY OF STRENGTHENED ELEMENTS

4.1 General procedure accounting for interface slip

The connecting procedure between the contact surfaces would have an important effect on the capacity of a composite element. Shear transfer at the interface is the most critical issue for an element subjected to bending. Figure 7 schematically presents the strain distribution against height of cross section for three different cases where strengthening has been carried out in the tensile region. The three different cases are as follows:

a) When the connection ensures no slippage between the contact surfaces, the behaviour of composite element can be considered as monolithic and the strain distribution with height of cross section will be linear, as shown in figure 7a,

b) When the connection allows free slippage between the contact surfaces, the behaviour of the composite element will be determined by the behaviour of the two parts of the element, as presented in figure 7b and

c) In practice, with realistic connections, the relative slippage between the contact surfaces depends on the magnitude of the shear stress that develops between the contact surfaces and the shear resistance. If there is slippage, then the strain distribution will have a discontinuity, as shown in figure 7c.

Obviously, the size of the discontinuity in the linear strain distribution (figure 7c) is dependent on the relative slippage between the contact surfaces (Lampropoulos and Dritsos, 2006; Tsioulou and Dritsos, 2006). Consequently, in order to not only estimate the strength of the composite element but also the amount of activated stress or developed deformation between the contact surfaces, a model to simulate the shear stress and the shear strain that would occur at the interface due to slippage would be required. This can be achieved by using a diagram of interface shear stress against slippage and would depend on the shear transfer mechanisms mobilised between the two parts of the element (as described in section 3.2 above and summarised by figure 6a). Simplified diagrams, as presented in figure 6b, can significantly minimise computation time.

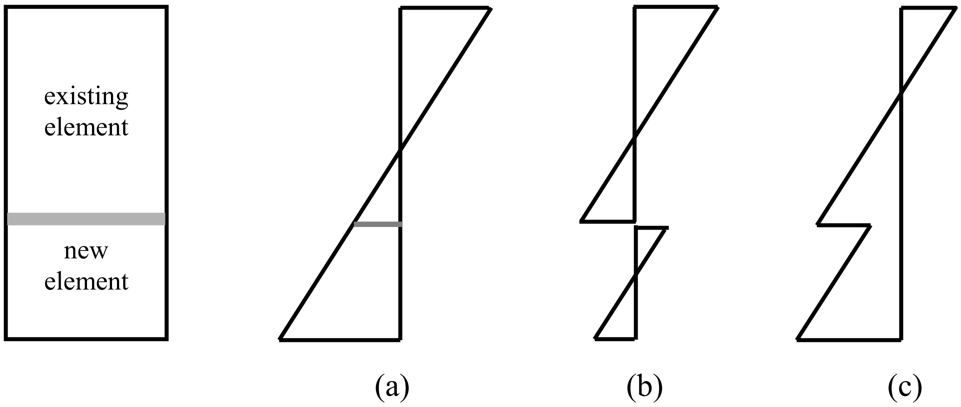


Figure 7. Distribution of strain with height of cross section for different connections between contact surfaces: (a) complete connection, (b) absence of connection and (c) partial connection.

In order to simplify calculations, further assumptions depending on the special conditions of the connection between the contact surfaces that could be considered, depending on the specific structural conditions, are as follows:

a) The thickness of the connection can be ignored and it can be considered as if the old and the new element are in complete contact,

b) The transfer of forces by adhesion can be ignored unless epoxy resin glue has been used between the contact surfaces and

c) When a glue, dowel or anchor has been used between the contact surfaces, only relative slippage at the interface can occur and it can be assumed that there will be no displacement perpendicular to the interface.

Consequently, when the element is subjected to bending, the curvature of two elements can be considered as the same. This simplification cannot always be applied. For example, when a new layer of concrete has been added to the original concrete element without an intermediary layer of resin or without shear connections, the simplification can only be considered when the surface of the original element has been mechanically scarified, roughened with a scabbier, sandblasted or water blasted and

d) The effects of shrinkage of the new material (for example, concrete or mortar) and the influence of differential creep could be ignored. The magnitude of the slippage that results between the contact surfaces should satisfy limit state controls for functionalism and failure, including those for which the effects of slippage are neglected.

The process of design for strengthened or repaired and strengthened elements described in this section can be extremely laborious and may involve a large calculation time. An analytical procedure has been proposed in the past (Dritsos, 1994; Dritsos, 1996) but any design process of this type is probably unfeasible in interventions of a large scale, unless suitable software has been developed. Moreover, in order to follow the above procedure, accurate and reliable models for the simulation of the contact surface behaviour are required and these models must take into account real on site working practices. To simplify calculations, an approximate process could be applied by first considering monolithic conditions in order to use well-known structural design methods for reinforced concrete elements. Therefore, the mechanical characteristics and the capacity of a strengthened or repaired and strengthened can be assessed by assuming the entire element, consisting of the old and additional elements, is monolithic. The results could then be corrected using special correction factors that can be defined as monolithic behaviour factors, for situations where reliable values are available.

4.2 Approximate procedure using monolithic behaviour factors

The transfer of the actual characteristics of response for the composite element may be considered equivalent to applying suitable corrective factors of simulation to a comparable monolithic element with the same characteristics and cross section. These monolithic correction factors for the stiffness (k_k) and the resistance (k_r) can be defined as follows:

$$k_k = \frac{(EI)_{Re}}{(EI)_{Mo}} \tag{12}$$

$$k_r = \frac{R_{Re}}{R_{Mo}} \tag{13}$$

where: $(EI)_{Re}$ and $(EI)_{Mo}$ are the stiffness of the retrofitted and corresponding monolithic element respectively and R_{Re} and R_{Mo} are the strength of the retrofitted and corresponding monolithic element respectively. The resistance indicator (r) may also individually concern bending capacity, shear capacity or axial force capacity and should be followed by indicators M, V or N respectively. Usually, k_k is less than or equal to k_r and k_r is less than or equal to unity.

Obviously, under the above framework, the complexity of the subject can be greatly simplified. With the given monolithic correction factors, the structural design and the assessment of the analysis data can be converted to the corresponding field of design for monolithic elements that is familiar to the engineer.

Modern seismic design of existing structures is moving towards a displacement capacity assessment of the structure. In this case, the yield strength or the failure strength of an element cannot be considered to fully express an element's capacity. In order to express the whole behaviour of an element, a capacity curve in terms of action effect against displacement is required. Figure 8 presents typical capacity curves for monolithic and strengthened elements. For monolithic elements, guidelines for design and recent draft Codes (FEMA 356, 2000; fib, 2003; GRECO, 2005) provide an idealized curve for any element, as shown in curve (a) of figure 8. Moreover, formulas are also provided to evaluate values for the yield and failure strengths ($F_{y,Mo}$ equals $F_{u,Mo}$) and for the displacement at both the yield stage ($\delta_{y,Mo}$) and at the failure stage ($\delta_{u,Mo}$) (fib, 2003; GRECO, 2005). However, appropriate formulas have not yet been made available in the literature regarding the corresponding relevant values for a strengthened element and any derived formulas cannot be verified due to limited experimental data.

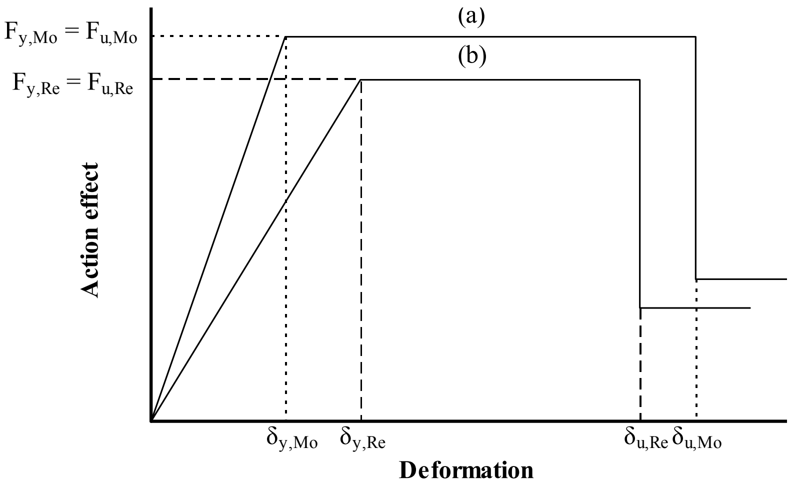


Figure 8. Capacity curves for (a) monolithic elements and (b) strengthened or repaired and strengthened elements.

Therefore, since the critical characteristics of an element are not only the strength and stiffness but also the displacement at the yield, maximum load and failure stages, additional monolithic behaviour factors regarding displacement capacities can be defined as follows:

$$k_{\delta y} = \frac{\delta_{y,Re}}{\delta_{y,Mo}} \quad (14a)$$

and
$$k_{\delta u} = \frac{\delta_{u,Re}}{\delta_{u,Mo}} \quad (14b)$$

where: $k_{\delta y}$ and $k_{\delta u}$ are the displacement monolithic behaviour factors at the yield and ultimate stages respectively and $\delta_{y,Re}$ and $\delta_{u,Re}$ are the displacements of the strengthened element at the corresponding stages.

If the above monolithic factors are provided (for example, by a Code), the capacity curve of the strengthened element, as in figure 8b, can be determined in terms of action effect against displacement. Consequently, for a flexure-controlled reinforced concrete element strengthened with additional reinforced concrete, the element's capacity curve could be determined in terms of bending moment and chord rotation by using the monolithic factor values k_r, $k_{\delta y}$ and $k_{\delta u}$, as follows:

$$M_{y,Re} = k_r * M_{y,Mo} \quad (15a)$$

$$\theta_{y,Re} = k_{\delta y} * \theta_{y,Mo} \quad (15b)$$

and
$$\theta_{u,Re} = k_{\delta u} * \theta_{u,Mo} \quad (15c)$$

where: $M_{y,Re}$, $\theta_{y,Re}$ and $\theta_{u,Re}$ express the capacities of the strengthened element regarding yield moment, chord rotation at yield and chord rotation at failure, while $M_{y,Mo}$, $\theta_{y,Mo}$ and $\theta_{u,Mo}$ express the respective capacities for the monolithic element.

The chord rotation (rather than plastic rotation) is a well understood mechanical characteristic used to express flexure-controlled deformation and has been adopted for European seismic design (Panagiotakos and Fardis, 2001; EC 8, 2004; fib, 2003; GRECO, 2005). The chord rotation is defined as the angle between the tangent to the axis of the element and the chord connecting the stressed section under consideration and the end of the shear span. In seismic design, the considered sections are normally the ends of the element and the chord rotations at the yield and ultimate stages can be calculated from semi-empirical expressions provided in many references in the literature (for example, fib, 2003; EC 8, 2004). The bending moment at yield can be easily calculated through either conventional design equations (CEB/fib, 1993) or other more accurate expressions provided by fib (2003).

Finally, the secant to yield stiffness of the strengthened element can be calculated from $M_{y,Re}$ and $\theta_{y,Re}$ without using the stiffness monolithic factor, as follows:

$$(EI)_{Re} = \frac{M_{y,Re}}{3\theta_{y,Re}} L_s \qquad (16)$$

where: L_s equals M/V and is the shear span of the strengthened element.

From the above discussion, it is obvious that the determination of reliable monolithic correction factor values, for use by the engineers in practice, is crucial for the application of the proposed redesign framework. Extensive analytic investigations and experimental trials are required in order to determine reliable correction factor values. In this direction, the real characteristics of stiffness, strength and displacement capacities of a retrofitted element must be evaluated. Then these characteristics must be compared with the characteristics of an equivalent monolithic element with the same cross sectional and detailing characteristics. Consequently, it is obvious that results will have an influence in practical applications only if the intervention can be applied in the same way as in the laboratory. However, it must be stress that, until now, no research project has been performed with the aim of producing monolithic factors for design purposes. Moreover, although there is some experimental data in the literature regarding strengthening by the addition of new reinforced concrete, appropriate data from which monolithic factors could be obtained is minimal. It is therefore evident that, in practice, the "engineer's judgement" will be required as, for many cases, the experimental data does not fit the specific application condition in practice. In any case, for practicing engineers, a useful guide to the estimate of monolithic correction factor values can be found in Part 1-4 of EC 8 (1995), fib Bulletin 24 (fib, 2003) and GRECO (2005). However, it must be stressed that values have been obtained in a more or less empirical way according to experts' knowledge and judgement and there is a lack of scientific justification. Experimental result based monolithic correction factor values for a number of interface connection procedures will be derived in the following section.

5. DERIVING VALUES FOR MONOLITHIC BEHAVIOUR FACTORS

For existing concrete frame buildings, designed only for vertical loads or designed under old seismic Code provisions, seismic retrofitting mainly concerns strengthening the vertical elements. If the technique of adding new reinforced concrete is chosen, this is mainly performed by placing reinforced concrete jackets around the old elements, as shown in figure 9. Sometimes, depending on the specific site conditions, the new concrete layer may be placed on one or more sides of the element.

In the following, monolithic correction factor values will be derived by processing available experimental data. For concrete jackets, experimental results from a number of research projects performed at the University of Patras (Bousias *et al.*, 2004; Vandoros and Dritsos, 2006a; Vandoros and Dritsos, 2006b; Vandoros and Dritsos, 2006c) will be processed and further analysed. For strengthening by the addition of new concrete layers, experimental data will be drawn from a past research project performed at the National Technical University of Athens (Vassiliou, 1975).

Figure 9. Concrete column before placing a new concrete jacket.

5.1 Concrete jackets

Figure 10 presents experimentally obtained force against displacement envelope curves concerning eight strengthened column specimens and a respective monolithic specimen (Mo) tested under cyclic loading. The respective envelope curve of an initial original column specimen (O) is also presented in the same figure for comparative purposes. In order to obtain values for the strength and the deformation at the three critical stages of yield, maximum load and failure load, trilinear idealizations of the experimental curves can be performed. The yield point can be determined by using a procedure similar those proposed by Pauley and Priestley (1992), ATC 40 (1996) and GRECO (2005). As shown figure 11a, the elastic branch of the trilinear curve crosses the real experimental curve at the point that is 60% of the yield load. In addition, up to the point of maximum load, the area created by the real experimental curve inside the trilinear curve must equal the area created by the real experimental curve outside the bilinear curve. The failure point is defined as the point on the descending branch of the experimental curve where the load is 20% less than the maximum lateral load. Figure 11b presents average values for the positive and negative quadrants of the trilinear curves. Eight original columns, with an initial cross section of 250 mm by 250 mm, were strengthened by placing a 75 mm thick shotcrete jacket. A local contractor was used for the work. Four 14 mm diameter grade S220 longitudinal reinforcing bars were placed at the corners of the original columns. This was equivalent to a reinforcement ratio of almost 1.0%, which is typical of old construction practice since the minimum specified in old Greek codes was 0.8%. The stirrups consisted of 8 mm diameter grade S220 bars spaced at every 200 mm. The longitudinal reinforcing bars of the jacket were four 20 mm diameter grade S500 bars placed at the corners of the jacket. Stirrups of 10 mm diameter grade S500 were placed in the jacket at a spacing of 100 mm. Thus, the reinforcement ratio of the jacketed column was 1.15%, which is within the range typically used for new construction. The monolithic column had exactly the same reinforcement and final cross sectional dimensions (400 mm by 400 mm) as the jacketed columns.

Four different treatment procedures were used at the interface between the original column and the jacket. An interface treatment (denoted as R) was used for columns R1 and R2 and involved the use of a mechanical scabbler to roughening the surface of the original column before strengthening. An interface treatment (denoted as D) was used for columns D1 and D2 and involved the use of dowels as connectors at the interface between the original column and the jacket. A combination of the above two interface treatments (denoted as RD) was used for columns RD1 and RD2. Finally, an interface treatment (denoted as W) was used for columns W1 and W2 and involved welding bent down steel connectors between the longitudinal bars of the original column and the jacket.

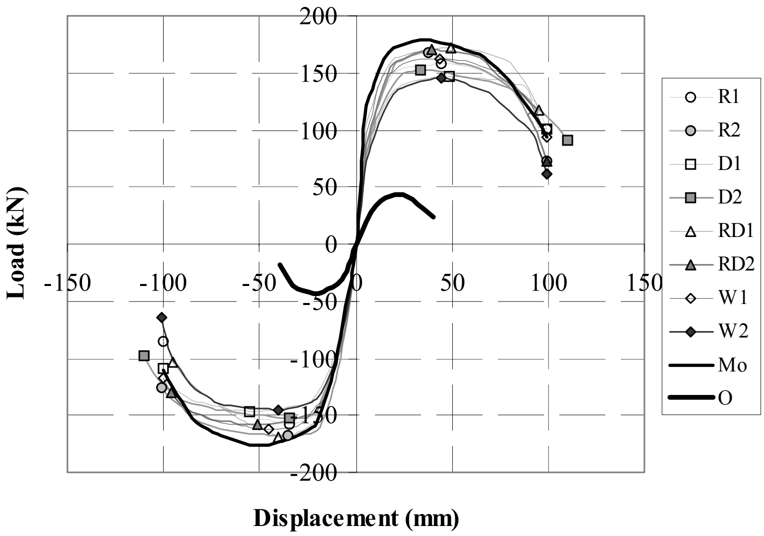
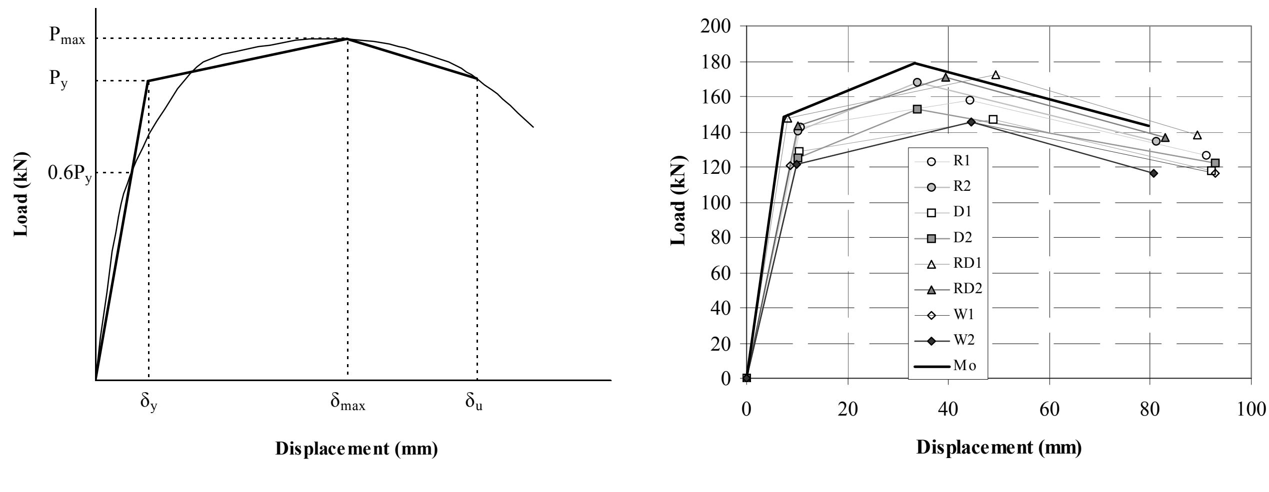


Figure 10. Load against displacement envelope curves for all tested specimens (Bousias et al., 2004; Vandoros and Dritsos, 2006b; Vandoros and Dritsos, 2006c).

The concrete strengths and maximum applied axial loads of the specimens are presented in table 1. As can be seen on table 1, high jacket strengths were achieved in most cases. This is common in practice as it satisfies the reasonable demand of minimizing the jacket thickness. Moreover, high concrete strengths can be easily achieved when shotcreting.



(a) (b)

Figure 11. Trilinear idealization curves: (a) definition and (b) results for the strengthened and monolithic specimens.

When determining the values of normalized axial load (v) for the strengthened specimens, the following formula was adopted:

$$v = \frac{N}{(A_{co} f_{co} + A_{cj} f_{cj})} \quad (17)$$

where: N is the applied axial load, and $A_{co}f_{co}$ and $A_{cj}f_{cj}$ are the cross sectional areas multiplied by the concrete strengths of original concrete and jacket respectively.

Table 1. Concrete strengths and applied axial loads for the experimental column specimens.

Specimen	Column concrete strength (MPa)	Jacket concrete strength (MPa)	Axial load (kN)	Normalized axial load
R1	27.0	55.8	930	0.130
R2	27.7	55.8	870	0.121
D1	27.0	55.8	920	0.129
D2	27.4	55.8	870	0.122
RD1	27.0	55.8	970	0.136
RD2	26.3	55.8	1050	0.148
W1	22.9	18.8	830	0.255
W2	22.9	18.8	790	0.242
Mo	24.7	-	1050	0.265
O	27.0	-	690	0.409

As it is evident from table 1, the reported values are not the same for every specimen. Consequently, a direct comparison between the strengthened columns and the monolithic specimen to identify the influence of the treatment at the interface is not possible, since the concrete properties and axial load significantly influence the results. Therefore, the experimental force against displacement curves have been analytically transformed as if hypothetical specimens with the same concrete strength and axial load as the monolithic specimen were under investigation. Vandoros and Dritsos (2006a) have previously presented the conversion procedure in detail. In summary, the conversion procedure consists of the following three steps:

a) For specimen Mo and for every strengthened specimen (considered as monolithic for this procedure), the load and displacement at the yield, the maximum load and the failure stages were analytically calculated using appropriate formulas (EC 2, 2004; EC 8, 2004).

b) Conversion coefficients were derived as the ratio of the analytical load and displacement values of specimen Mo at the above three critical stages to the corresponding values for each strengthened specimen and

c) Transformed values presented in figure 12 were obtained by multiplying the experimental values of figure 11 by the corresponding conversion coefficients.

Obviously, the concrete strength of the jacket and especially the normalized axial load have the greatest influence on the deformation capacities at the ultimate and failure stages. Thus, the largest differences between the initial and the transformed trilinear curves are observed for specimens R1, R2, D1, RD1 and RD2.

However, since concrete strength and especially the normalized axial load have the greatest influence on the deformation capacities at the ultimate and failure stages, the largest differences between the initial and transformed trilinear curves are observed for specimens R1, R2, D1, RD1 and RD2.

The transformed curves are presented in figure 12. By using equations 12 and 13, resistance monolithic factor values can now be derived for the yield ($k_{r,y}$), maximum ($k_{r,max}$) and ultimate ($k_{r,u}$) stages, as presented in table 2. Table 2 also presents the corresponding displacement monolithic factor values. The monolithic behaviour factor at the maximum load stage ($k_{\delta max}$) can be defined in a similar way to equations 14a and 14b as the displacement corresponding to the maximum load of the strengthened element ($\delta_{max,Re}$) divided by the displacement corresponding to the maximum load of the monolithic element($\delta_{max,Mo}$).

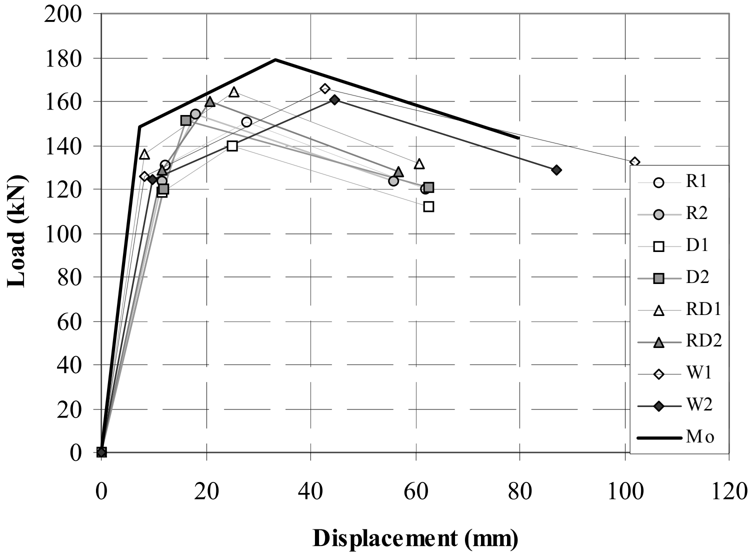


Figure 12. Transformed curves for the strengthened tested specimens and the monolithic specimen.

Table 2. Experimental monolithic factor values for RC jacketed columns.

Specimen	$k_{r,y}$	$k_{\delta y}$	$k_{r,max}$	$k_{\delta max}$	$k_{r,u}$	$k_{\delta u}$
R1	0.884	1.95	0.841	0.832	0.841	0.778
R2	0.835	1.85	0.863	0.543	0.863	0.701
D1	0.797	1.87	0.782	0.751	0.782	0.788
D2	0.808	1.89	0.843	0.489	0.843	0.786
RD1	0.918	1.32	0.917	0.758	0.917	0.762
RD2	0.868	1.84	0.892	0.625	0.892	0.714
W1	0.848	1.34	0.926	1.28	0.926	1.28
W2	0.840	1.58	0.897	1.34	0.897	1.09

However, for design applications two sources of uncertainties should be taken into consideration. These are: (a) differences between experiment laboratory results and what can be achieved by the real application in practice and (b) the models and assumptions used in the analytical work when transforming the force against displacement curves. Moreover, although when considering strength, it is conservative to accept the lowest possible values, this is not the case when deformation is considered. For this situation, average values for deformation, at the yield and failure stages, appears to be more accurate. Following on from these considerations, values for monolithic factors based on experimental results ($k_{r,exp}$, $k_{\delta y,exp}$ and $k_{\delta u,exp}$) that can be provisionally suggested for design purposes are presented in table 3. Values for k_k are not provided in table 3, however the secant of yield stiffness, for use in conventional elastic analysis, can be obtained by substituting equations 15a and 15b into equation 16, as follows:

$$(EI)_{Re} = \frac{k_r}{k_{\delta y}} \frac{M_{y,Mo}}{3\theta_{y,Mo}} L_s \qquad (18)$$

therefore,

$$(EI)_{Re} = \frac{k_r}{k_{\delta y}} (EI)_{Mo} \qquad (19)$$

and obviously, from equation 12 above

$$k_k = \frac{k_r}{k_{\delta y}} \qquad (20)$$

Table 3. Proposed design monolithic factor values for columns strengthened with concrete jackets under the construction and loading conditions used in the experimental work

Type of interface treatment	$k_{r,exp}$	$k_{\delta y,exp}$	$k_{\delta u,exp}$
R	0.80	1.90	0.75
D	0.77	1.85	0.80
RD	0.85	1.55	0.75
W	0.85	1.45	1.15

As it can be seen from table 3, the values for $k_{r,exp}$ are always lower than unity and range from 0.77 to 0.85, values for $k_{\delta y,exp}$ are far higher than unity and range from 1.45 to 1.90 while values $k_{\delta u,exp}$ range from 0.75 to 1.15 and, in most cases, are lower than unity.

Furthermore, it can be seen that the type of interface treatment influences the deformation factors $k_{\delta y,exp}$ and $k_{\delta u,exp}$ much more than the strength factor $k_{r,exp}$. In other words, the existence of steel connectors at the interface has a slight influence on strength but a considerable influence on the deformation capacity of the element. When comparing the different interface treatment procedures, specimens RD and W performed closer to monolithic behaviour. By taking into account that, for the experimental program, procedure W was performed by welding bent down bars in smooth interface conditions, behaviour even closer to monolithic could be expected if roughening had also been performed. However, procedure W is not usually undertaken on site, since it is a time consuming technique with a number of practical problems. These are: (a) the existing steel bars may be corroded and/or not weldable, (b) bend down bars must be individually detailed for each column to fit well between the old and the new bars and (c) welding on site can only be performed under specific conditions and only by qualified specialist welders. Therefore, values for monolithic factors regarding the RD procedure is of specific interest since, in practice this should be the recommended procedure for connecting at the interface, while it not acceptable to construct jackets without taking any special connecting measure between the old element and the jacket (Dritsos, 2005b; GRECO, 2005; Vandoros and Dritsos, 2006c).

Values of 0.80 for k_r and 0.70 for k_k are proposed in Part 1-4 of EC 8 (1995). When compared to the values for procedure RD from table 3, it could be concluded that these values are reasonably conservative for strength and slightly high for stiffness. The relevant values proposed for design in the recent draft version of Part 3 of EC 8 (2004) are in the range of 0.90 to 1.00 for k_{ry}, 1.05 to 1.25 for $k_{\delta y}$ and equal to 1.00 for $k_{\delta u}$. By comparing these values with the relevant ones presented in table 3, it is obvious that the proposed Code values are in reasonable agreement for the case of the strength coefficient k_r. This is not the case for displacement factors as values for $k_{\delta y}$ are underestimated and values of $k_{\delta u}$ are, in most cases, overestimated.

It must be stressed that analytical results regarding action effects in the vertical elements of a structure, when concrete jackets have been used to strengthen some columns, are not always on the side of safety if the lowest possible value for stiffness is used for the jacketed vertical elements. In this case, it is possible that maximum action effects and stresses can be underestimated in the vertical unstrengthened elements. In all cases, a conservative design for the existing elements of a structure would take into account the most unfavourable combination of action effects that would result from one of two analyses. In the first analysis, the lowest possible value of stiffness of the strengthened vertical elements should be considered by using either equation 17 or by completely ignoring the material of the original column. That is to say, by only taking into consideration the cross section of the jacket. In the second analysis, the stiffness of columns can be estimated by assuming a complete connection between the jacket and the original column. In other words, the entire cross section can be considered as monolithic.

It must also be stressed that values in table 2 and the proposed values in table 3 are limited by the specific parameters involved when obtaining the experimental data. Therefore, more experimental results are required, based on different element properties, dimensions and applied axial loads, before accurate values for monolithic factors can be generally proposed for design and Code use.

Alternatively, in the framework of the present study, analytical finite element simulations can be used to bridge the gap between experimental obtained values and design needs. To this end, the finite element program ANSYS (2002) has been used to perform analytical parametric studies that simulated the relative slippage between the contact surfaces, using special contact elements depending on the treatment procedure used at the interface. Economou (2004) investigated the main parameters affecting monolithic factor values and reported that, for typical practical applications, differences in the quantity of reinforcement had a minimal influence. Similarly, in the same research, the influence of the jacket concrete strength was found to have an insignificant effect. Two graphs from Economou (2004) are presented in figure 13 and justify the jacket concrete strength finding.

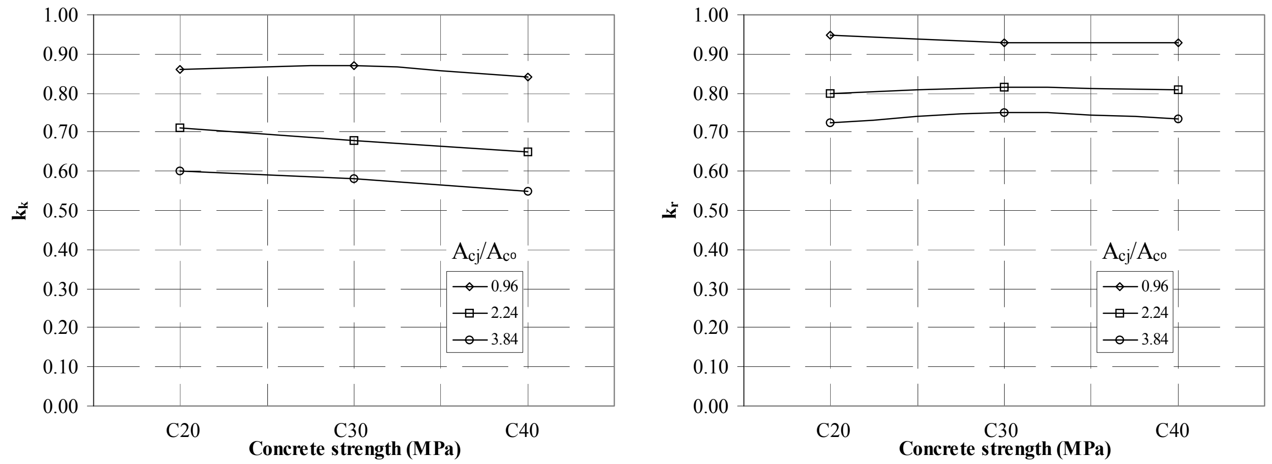


Figure 13. Concrete strength influence on monolithic values (Economou, 2004).

The critical parameters affecting the monolithic factor values were found to be: (a) the interface roughness, which can be expressed by the magnitude of the coefficient of friction (μ), (b) the value of the normalized axial load and (c) the ratio of cross sectional area of the jacket divided by the initial column cross sectional area (A_{cj}/A_{co}). Therefore, these three parameters are examined hereafter. A large number of hypothetical type R specimens with different combinations of the above parameters have been analysed. Best-fit curves for all examined simulations are presented in figures 14, 15 and

16. The plots of these figures have been normalised with respect to a reference specimen with μ, v and the ratio A_{cj}/A_{co} equal to 1.5, 0.125 and 1.56 respectively, since these values are the respective values for the experimental specimen R. To facilitate design purposes in conjunction with experimental values, plots have been drawn regarding the ratios $k_r/k_{r,exp}$, $k_{\delta y}/k_{\delta y,exp}$ and $k_{\delta u}/k_{\delta u,exp}$. From figures 14, 15 and 16, it can be seen that the capacity factor k_r increases when the coefficient of friction increases and the normalized axial load and the ratio A_{cj}/A_{co} decrease. Exactly the opposite can be observed when considering the deformation factors $k_{\delta y}$ and $k_{\delta u}$. Moreover, it can be seen that deformation factors $k_{\delta y}$ and $k_{\delta u}$ are more sensitive to the above parameters when compared to the capacity factor k_r.

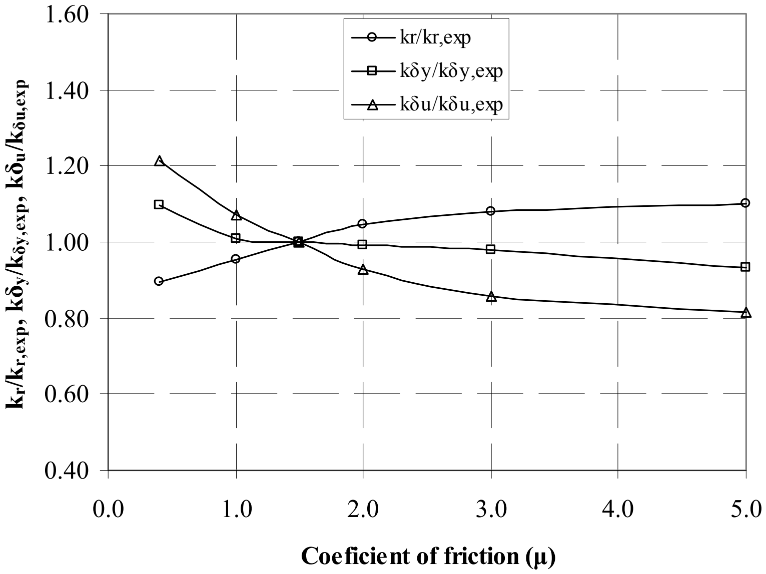


Figure 14. Monolithic factors against friction coefficient for concrete jacketing.

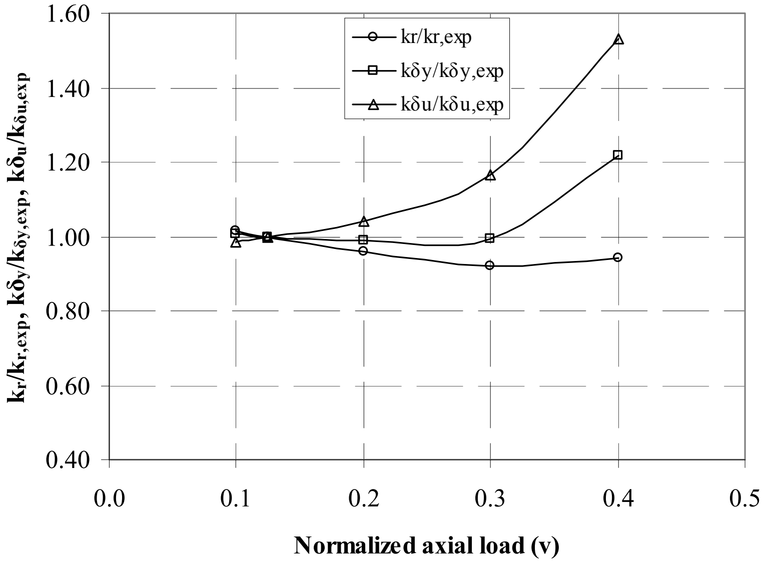


Figure 15. Monolithic factors against normalized axial load for concrete jacketing.

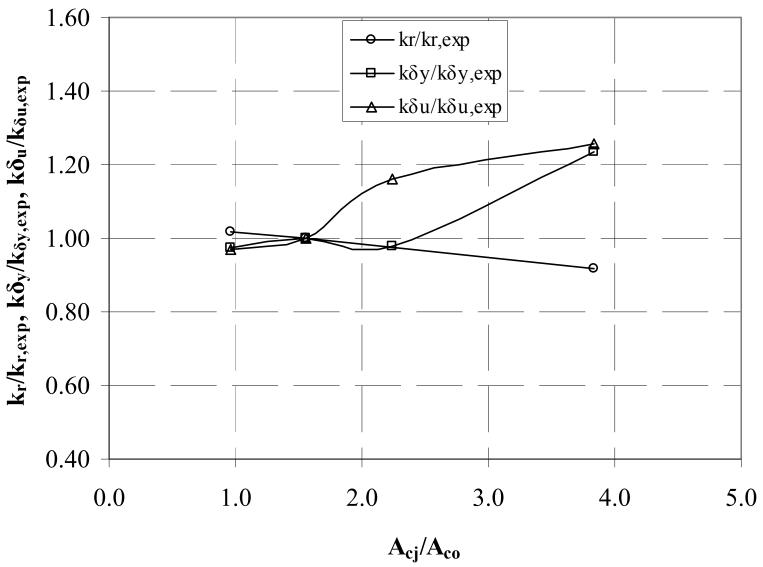


Figure 16. Monolithic factors against A_{cj}/A_{co} for concrete jacketing.

5.2 Concrete layers

Strengthening by placing concrete layers can be defined as the technique of placing concrete on one or more sides of an element. The procedure is worth investigating as the new concrete does not surround the element and a different behaviour can be expected when compared to strengthening by placing concrete jackets. Although experimental data exists for placing new concrete layers, suitable data for deriving monolithic factors is very rare in the literature because a corresponding monolithic element has not been presented. Vassiliou (1975) has performed an extended experimental work on beams and slabs strengthened by new concrete layers. However, monolithic factors can only be derived for slabs from this work, as the author did not test a relative monolithic beam. Vassiliou (1975) performed tests on 100 mm thick slabs strengthened by placing a new 30 mm thick layer on the compressive side of the slabs. From the experimental data, values of 0.94, 1.15 and 1.00 can be derived for k_r, $k_{\delta y}$ and $k_{\delta u}$ respectively. Similar values are proposed in Part 1-4 of EC 8 (1995) and GRECO (2005) for the design of beams. Both publications propose values of 0.90 for k_r and 0.85 for k_k. However, EC 8 (1995) and GRECO (2005) assume full monolithic behaviour for slabs, that is, k_r, k_k, $k_{\delta y}$, and $k_{\delta u}$ all equal unity.

Adopting the above range of values for slabs and beams, the question comes to columns. When considering the different loading conditions of slabs, beams and columns, the influence of axial load on monolithic factors should be examined. Since relevant experimental data does not exist in the literature, the finite element programs ANSYS (2002) and ATENA (Cervenka et al. 2005) have been used to perform parametric studies concerning the influence of the normalized axial load on monolithic factors values.

For this study, hypothetical specimens were considered that had a 250 mm by 250 mm original column cross section and were strengthened by placing a new 75 mm thick concrete layer. Therefore, the ratio of new layer thickness to original concrete thickness was the same as for the experimental specimens tested by Vassiliou (1975). Moreover, in order to match with the results obtained by the experimental work of Vassiliou (1975), a typical roughening of the surface of the original element was assumed for the interface treatment. Therefore, the coefficient of friction was considered to equal 1.5 (this value is normally applied for a properly executed typical roughening).

From the analytical data, it was found that the magnitude of the axial load does not play an important role on the displacement factor values $k_{\delta y}$ and $k_{\delta u}$, while it significantly influences the k_r strength factor values. Figure 17 presents the best-fit curve concerning values of the monolithic factor k_r, for all analytical specimens examined. In figure 16, the factor values for k_r under zero axial load is denoted as $k_{r,o}$. From figure 17, it is obvious that the value of monolithic factor k_r rapidly decreases as the axial load increases. Furthermore, when considering a typical column with a normalized axial load in the order of 0.4, it appears that the capacity of a strengthened column is roughly half that of the respective monolithic (as $k_{r\,(v = 0.4)} = 0.60{*}k_{r,\,o} = 0.60{*}0.90 = 0.54$). Thus, it seems that in typical loading situations, it is ineffective to strengthen concrete columns by using a type R interface treatment procedure when applying concrete layers.

Acknowledging the reputable effectiveness of a type W interface treatment procedure when strengthening by adding jackets (see table 3), further analytical work has been performed for the case of adding concrete layers following the above procedure. Figure 18 presents analytical results using ATENA software (Cervenka *et al.* 2005) for two characteristic strengthened specimens. Results are presented for specimens with only roughening at the interface or only bend down bars at the interface. In addition, results for the respective monolithic column and the original column are presented for comparison reasons. The same normalized axial load, corresponded to a value 0.3, was applied for specimens R, W and M and a normalized axial load of 0.4 was applied for specimen O. The beneficial action of the bend down bars can be appreciated from figure 18. It is obvious that the use of steel connectors at the interface is essential.

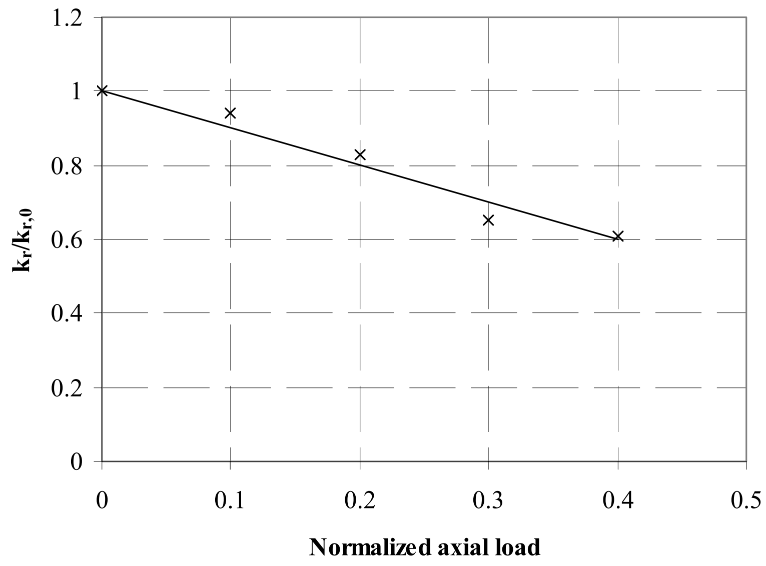


Figure 17. Monolithic factor against normalized axial load for the addition of a concrete layer.

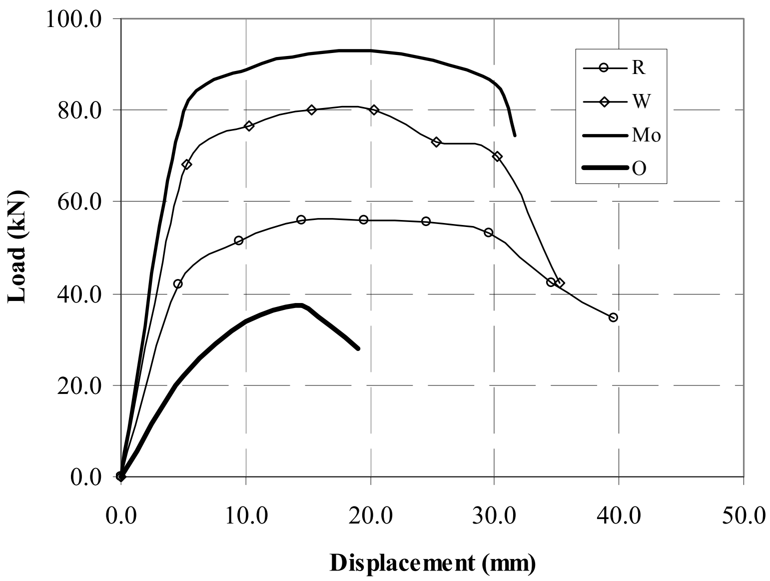


Figure 18. Load against displacement analytical results.

Lastly, it should be recognized that monolithic factors described above for the case of strengthening by new concrete layers do not have enough justified experimental supporting evidence. Much more experimental work is required to derive values that are more accurate. However, since this type of work is often executed in retrofitted works in practice and acknowledging that there is an almost complete absence of guidance for the engineer, the derived values are provisionally proposed based on the above mentioned data in order to cover the practical design needs of today.

6. CONCLUSIONS AND MAIN OBSERVATIONS

Strengthening or repairing and strengthening columns by the addition of reinforced concrete jackets or layers are normal construction practices in many earthquake prone countries. However, there are many unresolved issues regarding the capacity of the strengthened elements. Engineering judgement is often used in place of any other guidance. This paper has set out to assist the engineer when considering some of these unresolved issues. To this end, the following conclusions and main observations can be drawn from the subject of the design of RC elements strengthened by the addition of new concrete:

1. The structural design of strengthened concrete elements can be placed in the framework of the presently known processes of design that are used for monolithic elements, supplemented by the following: (a) the use of revised factors of safety, (b) the control of a sufficient connection between contact surfaces and (c) the determination of the performance of the strengthened elements considered as “composite” elements,

2. For an existing rather than a new structure, revised safety factor values have been proposed and should be adopted when assessing the existing materials, existing dead loads and new materials.

3. To guarantee a sufficient connection between the old and the new concrete, the internal forces acting at the interface should not exceed the corresponding internal capacities,

4. The control of a sufficient interface shear capacity has been recognised as the most critical factor to avoid a premature failure,

5. Interface shear forces should be evaluated by adopting a procedure similar to that used for structural steel and concrete composite elements,

6. It has been recognised that there are four possible main mechanisms for the transfer of shear stresses acting at the interface. These are adhesion, friction (including clamping action), steel connectors and bent down bars. Simplifications for the superposition of the four main acting mechanisms have been proposed,

7. The evaluation of capacity of a strengthened element has been presented through the assessment of the slip distribution along the element's interface between the old and the new concrete,

8. An approximate procedure has been proposed based on the idea that analysis data from the field of monolithic design familiar to the engineer can be used, supplemented by the use of monolithic behaviour factors, to evaluate the capacity of a strengthened element,

9. Monolithic behaviour factor values have been derived by analysing available experimental data and by performing an extended analytical work to fill in the gaps in the experimental data,

10. From the analytical work, it was found that the resistance capacity factor, k_r, increases when the interface roughness increases and the normalized axial load and the ratio of the cross sectional area of original concrete and the jacket decrease. Exactly the opposite occurs when considering the yield and ultimate deformation factors $k_{\delta y}$ and $k_{\delta u}$. In addition, it was found that the deformation factors $k_{\delta y}$ and $k_{\delta u}$ were more sensitive to the above parameters when compared to the capacity factor k_r,

11. The beneficial action of the bend down bars has been recognised and is obvious that the use of steel connectors at the interface is absolutely necessary when placing additional concrete layers and

12. The monolithic factors described above for the case of strengthening by new concrete layers do not have enough justified experimental supporting evidence. Further experimental work is required before deriving more accurate values. However, since this type of work is often executed in retrofitted works in practice and acknowledging that there is an almost complete absence of guidance for the engineer, the derived values are provisionally proposed based on existing data in order to cover the practical design needs of today.

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